2 Theoretical Basis for SAM.hyd Calculations

Purpose

The purpose of the hydraulics module SAM.hyd is to provide hydraulic design engineers with a systematic method, based on state of the art theory, for rapidly calculating channel size in both fixed and mobile boundary streams.

General

SAM.hyd solves the steady state, normal-depth equation to transform complex geometry into composite, 1-dimensional hydraulic parameters. The program can solve for either depth, width, slope, discharge, or roughness. It allows roughness to be calculated by several different roughness equations within the same cross-section.

SAM.hyd provides the option of calculating riprap size as described in EM 1110-2-1601, "Hydraulic Design of Flood Control Channels" (USACE 1991, 1994) or through testing the results of the normal depth calculations against the Shield's Diagram for particle stability. The 13 standard riprap sizes from EM 1110-2-1601 are coded into SAM. The user can also specify quarry run riprap.

SAM.hyd offers two procedures for calculating riprap. When flow velocity and depth are known, the riprap size is calculated directly. When water discharge and cross section are given, riprap size is determined by a more complicated calculation since n-value becomes a function of riprap size.

SAM.hyd also has the ability to determine whether or not riprap is required. The program evaluates the calculated bed shear stress and the Shield's critical value. If the calculated bed shear stress is greater than Shield's critical value, SAM.hyd will notify the user that riprap is required. At this point the user may alter the data file to request riprap calculations.

Regime channel dimensions, determined using the Blench (1970) equations, can be calculated as a function of bed material gradation, bank consistency, and bed-material sediment concentration. SAM.hyd also contains an analytical procedure for calculating stable channel dimensions. It is based on the Brownlie equations for sediment transport and bed roughness. This calculation provides a table of channel width-depth-slope dimensions which are in equilibrium with the inflowing sediment load. Channel dimensions at minimum stream power are also calculated.

SAM.hyd Options

SAM.hyd can solve for any one of the variables in the uniform flow equation. Water discharge is usually the dependent variable (equation 2-1). However, SAM allows any of the variables on the right hand side to become the dependent variable except side slope, z. SAM.hyd will inspect each input record type in a data set and determine which variables have been prescribed. The variable omitted becomes the dependent variable.

$$Q = f(D, n, W, z, S)$$ **Equation 2-1**

where
- Q = water discharge
- D = water depth
- n = n-value
- W = bottom width
- z = side slopes of the channel
- S = energy slope

Most of the major calculation options in SAM.hyd rely on the above relationship. An overview of these major options follows.

Normal Depth	will be calculated when all variables except water surface elevation are prescribed.
Bottom Width	of the channel will be calculated when all variables except bottom width are prescribed.
Energy Slope	will be calculated when all variables except slope are prescribed.
Hydraulic Roughness	(n-values and k_s values) will be calculated when all variables except n-value are prescribed.
Water Discharge	will be calculated when all variables except water discharge are prescribed.

Flow Distribution will be calculated when all variables are prescribed. Flow distribution is also calculated each time the uniform flow equation is solved, and can be requested during any of the calculations.

An overview of other SAM.hyd calculations follows.

Composite Hydraulic Parameters can be calculated by four options: alpha method, equal velocity method, total force method and conveyance method.

Bed Form Roughness Prediction is made using Brownlie's (1983) method. It uses velocity, hydraulic radius, slope, particle specific gravity, d_{50} and the geometric standard deviation of the bed sediment mixture.

Cross Section Velocity is printed for comparison with the velocity criteria for stable channel design.

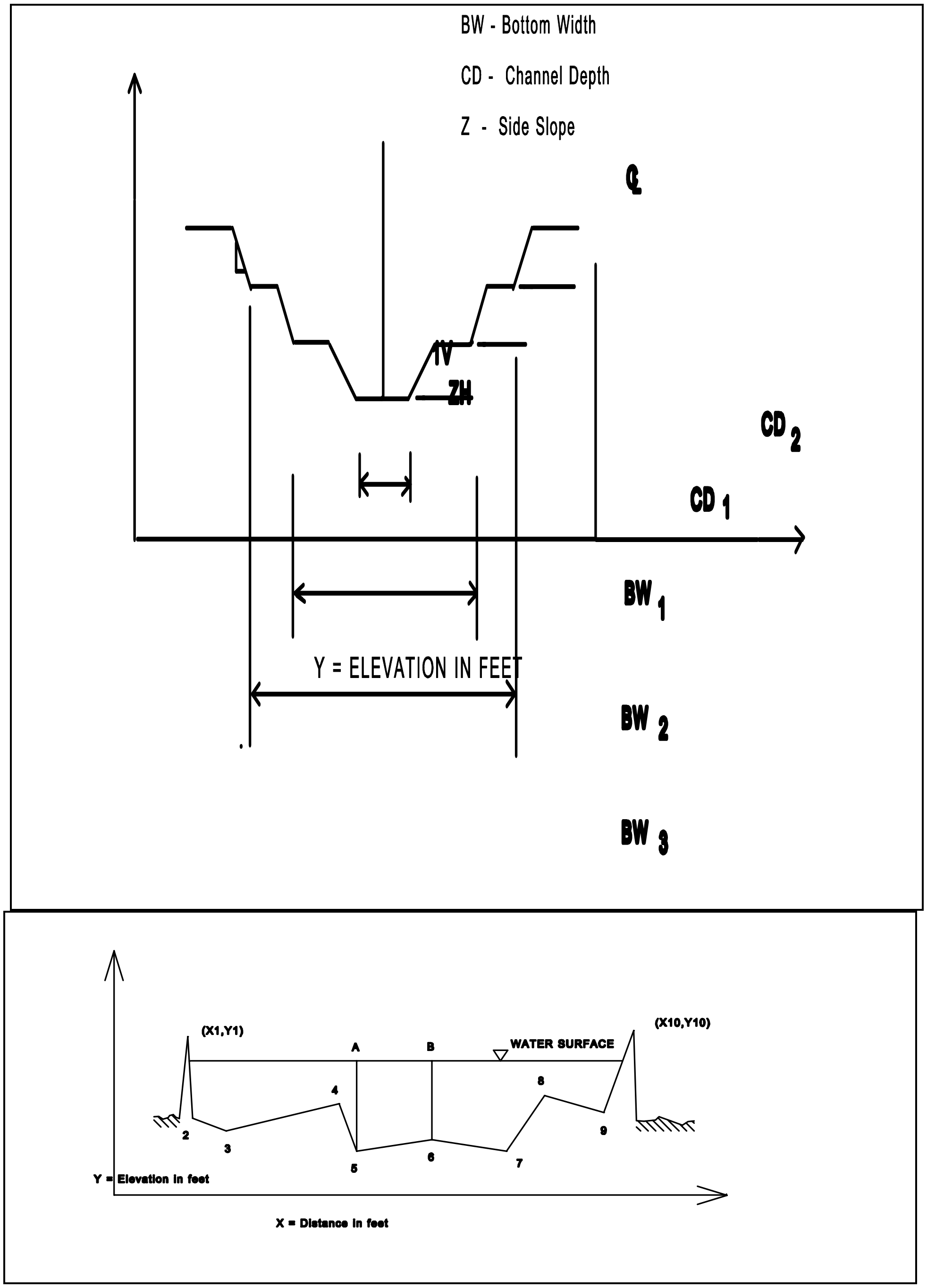
Shear Stress is printed for comparison with the boundary shear stress criteria for stable channel design.

Riprap Size can be requested. If the calculated bed shear stress is greater than Shield's critical value, the program will notify the user.

Effective Width, Depth and Velocity are calculated, printed, and written to an input file for use in sediment transport calculations, SAM.sed.

Equivalent Hydraulic Radius and n-Value are calculated and printed for each subsection in the cross section after the normal depth calculations are completed.

Geometry There are two options for coding the cross section. Single or compound channels can be prescribed by defining the bottom width and side slopes for simple triangular, rectangular or trapezoidal shapes. For example, Figure 1A shows a low flow channel, a normal flow channel, and a high flow berm. Complex channels can be prescribed by defining the station and elevation coordinates, as on X1 and GR records in HEC-2/HEC-6. Figure 1B shows a typical complex cross section as an example of what could be coded in this format.



NOTE: The trapezoidal element 'AB(6)(5)' in this figure is called a "panel" in this document.

Figure 1a and 1b. Complex Geometry using CT records and using X1 and GR records.

Normal Depth Calculations

Normal depth is calculated using one of five uniform flow equations, or one of five USDA Soil Conservation Service (SCS) equations for grass-lined channels. Different equations may be used in different panels. An iterative procedure is used to converge on the specified total discharge. Then a composite Manning's roughness coefficient and effective hydraulic parameters are calculated for the cross section.

Manning Roughness Equation When the hydraulic roughness is prescribed as

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$ **Equation 2-2**

n-values, the Manning equation is used to determine normal depth,
where

V	=	velocity, feet per second
R	=	hydraulic radius, feet
S	=	slope, feet per feet
n	=	Manning's roughness coefficient

Hydraulic Roughness Equations Hydraulic roughness can be prescribed directly with n-values (option 0) or it can be related to the physical properties of the cross section by one of the other following hydraulic roughness equations:

0. Manning’s Equation
1. Keulegan equations
2. Strickler Equation
3. Limerinos Equation
4. Brownlie Bed Roughness Equations

5-9. Five Soil Conservation Service equations for grass lined channels

Table 1. Hydraulic Roughness Options in SAM.hyd.

Note that as each channel problem is unique, no one predictor should be considered as "best."

Effective Surface Roughness Height, k_s

A value for k_s is required when the Keulegan equations or Strickler equation is specified. For the design of concrete channels the suggested values for k_s are shown in EM 1110-2-1601. For the case of channels in natural materials, there are no tables of generally accepted k_s values as there are for Manning's n-values. Moreover, there is no generally accepted technique for measuring this property geometrically. Therefore, unless a specific value of k_s is known, it is recommended that the hydraulic roughness be prescribed with n-values or by another analytical method. When sufficient data are available -- discharge, area, hydraulic radius, and slope -- k_s can be calculated and then used to calculate hydraulic parameters for additional discharges.

Relative Roughness

Relative roughness refers to the ratio of the effective surface roughness height, k_s, to the hydraulic radius, R. The relative roughness parameter is R/k_s. When this parameter is less than 3, which indicates a very rough surface, the logarithmic velocity distribution theory breaks down. For this reason, SAM will not apply the Keulegan equation when relative roughness is less than 3. Instead, the Strickler equation is automatically substituted.

Keulegan equations, rigid bed

The relative roughness approach based on the Keulegan (1938) equations for velocity distribution is still the state of the art for rigid boundary channel design. Keulegan classified flow types as hydraulically smooth, hydraulically rough, or transitional. His equations can be combined with the Chezy equation to obtain a Chezy roughness coefficient. The equations are written in terms of the Chezy coefficient, C, because the powers are simpler than when the Manning's equation is employed. Chezy's C may then be converted directly to a Manning's n value.

(1) The equation for fully rough flow is

$$C = 32.6 \log_{10}\left(\frac{12.2\, R}{k_s}\right)$$ **Equation 2-3**

(2) For smooth flow the equation is

$$C = 32.6 \log_{10}\left(\frac{5.2\, R_n}{C}\right)$$ **Equation 2-4**

(3) The equation for transitional flow is

$$C = -\,32.6 \log_{10}\left(\frac{C}{5.2\, R_n} + \frac{k_s}{12.2\, R}\right)$$ **Equation 2-5**

(4) The equation showing the relationship of n-value and Chezy *C* is

$$n = \frac{1.486}{C} R^{1/6}$$ **Equation 2-6**

where

R_n = Reynolds number
= $4RV/\upsilon$
v = kinematic viscosity of water

and 32.6, 12.2 and 5.2 are empirical coefficients determined from laboratory experiments. These equations, when graphed, produce the Moody-type diagram for open channel flow, Figure B-3, EM 1110-2-1601.

The Iwagaki relationship

Using experimental data obtained from many sources, Iwagaki (Chow, 1959) found that the coefficients 12.2 and 5.2 in equations 2-3, 2-4, 2-5, and 2-6 varied with Froude number. The results of his study disclosed that resistance to turbulent flow in open channels becomes obviously larger with increase in the Froude number. Iwagaki reasoned that this is due to the increased instability of the free surface at high Froude numbers. The Iwagaki relationship is shown in Figure 2.

Keulegan's equations can be modified to incorporate Iwagaki's results.

(1) Chow (1959) presents Keulegan's equation for the average flow velocity *V* in the following form

$$V = U_* \left[6.25 + 5.75 \log_{10}\left(\frac{R}{k_s}\right) \right]$$ **Equation 2-7**

where

$U_* = \sqrt{gRS}$ = boundary shear velocity
g = acceleration of gravity
6.25 = empirical constant for fully rough flow

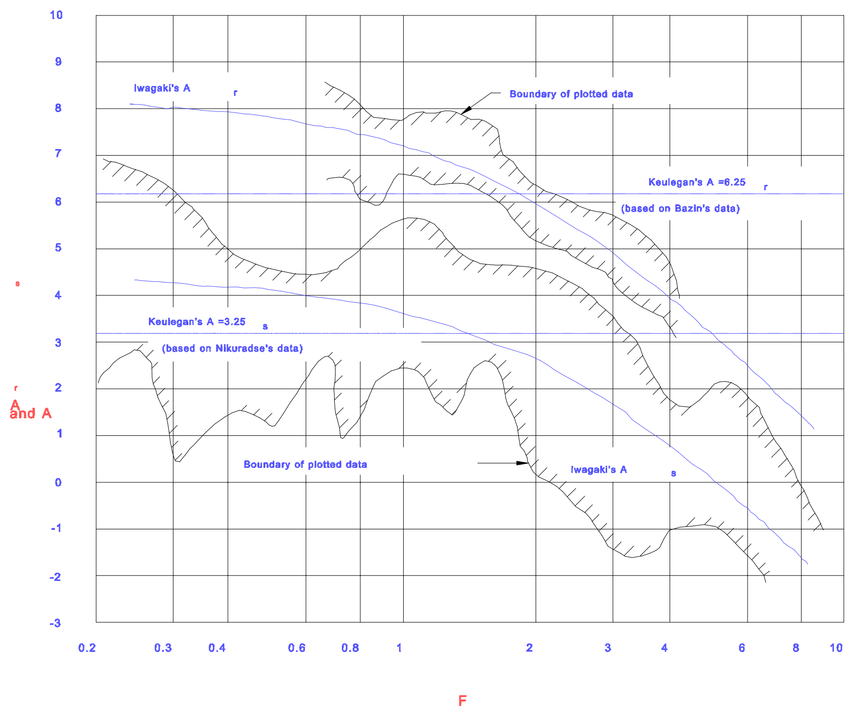


Figure 2. The Iwagaki relationship (Chow, 1959), used with permission from McGraw-Hill Book Company, Inc.

(2) The equation for rough flow may be obtained by substituting a variable, A_r, for the constant, 6.25, and $\sqrt{gRS}$ for U_*, and then combining with the Chezy equation.

$$\frac{V}{U_*} = \frac{C}{\sqrt{g}} = A_r + 5.75 \log_{10}\left(\frac{R}{k_s}\right)$$ **Equation 2-8**

$$C = \sqrt{g}\left[A_r + 5.75 \log_{10}\left(\frac{R}{k_s}\right)\right]$$ **Equation 2-9**

$$C = 32.6 \log_{10}\left[10^{\frac{A_r\sqrt{g}}{32.6}}\left(\frac{R}{k_s}\right)\right]$$ **Equation 2-10**

where A_r is the Iwagaki variable for rough flow. From Keulegan's study of Bazin's data, the value of A_r was found to have a wide range, varying from 3.23 to 16.92. Keulegan used a mean value of 6.25 for A_r.

(3) The comparable form of the equation for smooth flow is

$$C = 32.6 \log_{10}\left[10^{\frac{A_s\sqrt{g}}{32.6}}\left(\frac{\sqrt{g}\,R_n}{4C}\right)\right]$$ **Equation 2-11**

where A_s is the Iwagaki variable for smooth flow.

(4) The roughness equation in the transition zone is a combination of the equations for smooth and fully rough flow as follows:

$$C = -\,32.6 \log_{10}\left(\frac{4C}{\sqrt{g}\,R_n\,10^{\frac{A_s\sqrt{g}}{32.6}}} + \frac{k_s}{R\,10^{\frac{A_r\sqrt{g}}{32.6}}}\right)$$ **Equation 2-12**

A_r and A_s variables (1) The A_r and A_s variables are shown graphically in Figure 2. An equation for A_r is

$$A_r = -\,27.058 \log_{10}(F+9) + 34.289$$ **Equation 2-13**

where F is the Froude number. Data ranges from $0.2 < F < 8.0$.

(2) Using an equation of the same form, the relationship for A_s is

$$A_s = -\,24.739 \log_{10}(F+10) + 29.349$$ **Equation 2-14**

Equations 2-6, 2-10, 2-12, 2-13 and 2-14 are used in SAM.hyd to calculate the roughness coefficient when the Keulegan equations are specified.

Strickler Equation

The Strickler function is compared to measured data in Figure 3. (Chow, 1959) This figure shows that for a wide range in relative roughness, R/k_s, the variability of the Strickler function $\varphi\, R/k_s$ is small. Strickler assumed this to be a constant, 0.0342 when k_s and R were expressed in feet. The effective surface roughness height, k_s, is the d_{50} of the bed sediment in this figure. However, k_s can be correlated with other measures of the surface roughness, depending on what is representative of the surface roughness height of the boundary materials. For example, riprap research at WES (Maynord 1991, 1992) has shown that the Strickler equation will give satisfactory n-values when k_s is taken to be the d_{90} of the stone.

$$n = \phi \frac{R}{k_s} k_s^{1/6}$$ **Equation 2-15**

where, in SAM,

k_s = effective surface roughness, ft

$\varphi\, R/k_s$ = 0.0342 for natural channels,
where $k_s = d_{50}$
= 0.0342 for velocity and stone size calculations in riprap design,
where $k_s = d_{90}$
= 0.038 for discharge capacity calculations in riprap design,
where $k_s = d_{90}$

SAM uses different values for the Strickler function, depending on the calculations to be made. Chow (1959) recommends the use of 0.0342 for natural sediment, so that is the default value. In accordance with EM 1110-2-1601, 0.0342 is also used for velocity and stone size calculations. Thus, the same Strickler function is used in specified panels for cross-sections where some panels have riprap and others have natural sediment. Also in accordance with EM 1110-2-1601, 0.038 is used for water surface calculations after riprap has been calculated. In this case, n-values for all panels in the cross section will use this constant whether riprap was prescribed for that panel or not, possibly calculating some n-values incorrectly. Problems can be avoided by using another roughness equation in panels without riprap.

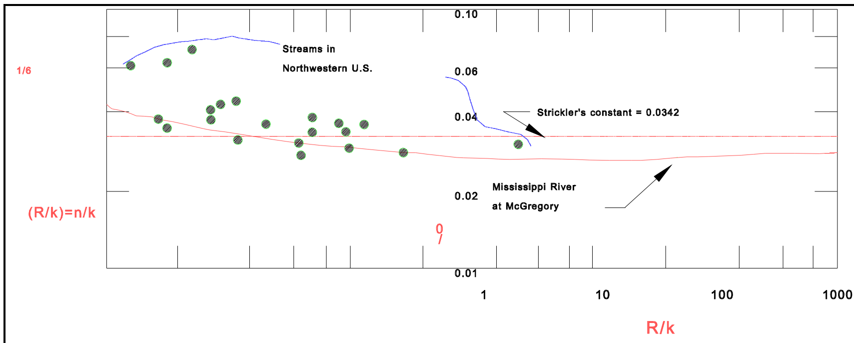


Figure 3. The Strickler Function (Chow, 1959), used with permission from McGraw-Hill Book Company, Inc.

Limerinos n-Value Predictor

Limerinos (1970) developed an empirical relative roughness equation for coarse, mobile bed streams using field data. He correlated n-values with hydraulic radius and bed sediment size. The resulting equation is shown below.

$$n = \frac{0.0926\ R^{1/6}}{1.16 + 2.0 \log_{10}\left(\frac{R}{d_{84}}\right)}$$

Equation 2-16

where

d_{84} = the particle size, ft, for which 84% of the sediment mixture is finer. Data ranged from 1.5 mm to 250 mm.
n = Manning's n value. Data ranged from .02 to 0.10.
R = Hydraulic radius, ft. Data ranged from 1 to 6 ft.

Grain sizes in Limerinos's data ranged from very course sand to large cobbles. Consequently, it follows that this equation is applicable to gravel/cobble bed streams and to bed regimes similar to those found in such streams and within the energy spectrum contained in Limerinos' field data. N-values predicted with the Limerinos equation are sufficiently larger than those predicted by the Strickler equation indicating that some loss other than grain roughness must have been present. The Limerinos equation is not applicable to lower regime flow nor does it forecast the transition between upper and lower regimes.

Burkham and Dawdy (1976) showed that the Limerinos equation could be used in sand bed streams provided the regime was plane bed. In their analysis they extended the range of the relative roughness parameter as follows.

$$600 < \frac{R}{d_{84}} < 10{,}000 \qquad \textbf{Equation 2-17}$$

Figure 4. Velocity versus hydraulic radius in a mobile bed stream, used with permission of California Institute of Technology.

Brownlie (1983) Bed-roughness Predictor

In sediment transport calculations it is important to link n-values to the bed regime. This is particularly true when hydraulic conditions shift between upper regime and lower regime flow.

Brownlie sought to reconstitute the discontinuity in the graph of hydraulic radius versus velocity, Figure 4. (Brownlie 1983) In the process of this research, he collected the known sediment data sets, 77 in all, containing 7027 data points. Of the total, 75 percent were from flume studies and 25 percent from field tests. He used 22 of these data sets and demonstrated a significant agreement with both field and laboratory data.

Brownlie's basic equations were modified for SAM to display bed roughness as a coefficient times the grain roughness. Any consistent set of units are applicable.

n = [Bedform Roughness] * [Strickler Grain Roughness] **Equation 2-18**

The resulting form of the equations for lower and upper regime are:

LOWER REGIME FLOW:

$$n = \left(1.6940 \left(\frac{R}{d_{50}} \right)^{0.1374} S^{0.1112} \sigma^{0.1605} \right) 0.034 \left(d_{50} \right)^{0.167}$$ **Equation 2-19**

UPPER REGIME FLOW:

$$n = \left(1.0213 \left(\frac{R}{d_{50}} \right)^{0.0662} S^{0.0395} \sigma^{0.1282} \right) 0.034 \left(d_{50}^{\ 0.167} \right)$$ **Equation 2-20**

where

d_{50} = the particle size for which 50% of the sediment mixture is finer.

R = hydraulic radius of the bed portion of the cross section.

S = bed slope; probably the energy slope would be more representative if flow is non-uniform.

σ = the geometric standard deviation of the sediment mixture, where

$$\sigma = 0.5\left(\frac{d_{84}}{d_{50}} + \frac{d_{50}}{d_{16}}\right)$$ **Equation 2-21**

TRANSITION FUNCTION:

If the slope is greater than 0.006, flow is always UPPER REGIME. Otherwise, the transition is correlated with the grain Froude number as follows:

$$F_g = \frac{V}{\sqrt{(s_s - 1)\, g\ d_{50}}}$$ **Equation 2-22**

$$F_g' = \frac{1.74}{S^{1/3}}$$ **Equation 2-23**

if $F_g < F_g'$ Lower Regime Flow

if $0.8\ F_g' < F_g < 1.25\ F_g'$ Transition Flow

if $F_g > F_g'$ Upper Regime Flow

where

F_g = grain Froude number.
s_s = specific gravity of sediment particles.
V = velocity of flow.
S = bed slope.

The transition occurs over a range of hydraulic radii and not at a point. Over this range, then, it is a double valued function, and the transition test will give different regimes depending on which equation is being solved for roughness at that iteration. That is realistic since one expects the rising side of a hydrograph to trigger the transition at a different discharge than does the falling side. This can cause the calculations to fail to converge in the transition zone, and the program will notify the user. If this occurs, inspect the maximum and minimum values printed. Usually, the depths are the same whereas the discharges are significantly different. This signifies that the transition has been located from both the upper regime and the lower regime curves, and the results are the best that can be achieved.

Soil Conservation Service (SCS) Roughness for Grass Cover

Hydraulic roughness curves for five types of grass cover were published by SCS (US Department of Agriculture 1954), Figure 5. Each curve type, A through E, refers to grass conditions described in table 4. EM 1110-2-1601 presents an example of the use of these SCS curves.

Table 4. Characteristics of Grass Cover.

Type	Cover	Condition
A	Weeping lovegrass................	Excellent stand, tall (average 30 in.)
	Yellow bluestem Ischaemum....	Excellent stand, tall (average 36 in.)
B	Kudzu	Very dense growth, uncut
	Bermudagrass......................	Good stand, tall (average 12 in.)
	Native grass mixture (little bluestem, blue grama, other long and short midwest grasses)	Good stand, unmowed
	Weeping lovegrass................	Good stand, tall (average 24 in.)
	Lespedeza serices.................	Good stand, not woody, tall (average 19 in.)
	Alfalfa..............................	Good stand uncut (average 11 in.)
	Weeping lovegrass...............	Good stand, mowed (average 13 in.)
	Kudzu..............................	Dense growth, uncut
	Blue grama........................	Good stand, uncut (average 13 in.)
C	Crabgrass..........................	Fair stand, uncut (10 to 48 in.)
	Bermudagrass.....................	Good stand, mowed
	Common lespedeza.............	Good stand, uncut (average 11 in.)
	Grass-legume mixture--summer (orchard grass, redtop, Italian ryegrass and common lespedeza)	Good stand, uncut (6 to 8 in.)
	Centipedegrass....................	Very dense cover (average 6 in.)
	Kentucky bluegrass..............	Good stand headed (6 to 12 in.)
D	Bermudagrass......................	Good stand, cut to 2.5-inch height
	Common lespedeza..............	Excellent stand, uncut (average 4.5 in.)
	Buffalograss.......................	Good stand, uncut (3 to 6 in.)
	Grass-legume mixture--fall, spring (Orchardgrass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (4 to 5 in.)
	Lespedeza sericea.................	After cutting to 2-inch height; very good stand before cutting
E	Bermudagrass......................	Good stand, cut to 1.5-inch height
	Bermudagrass......................	Burned stubble

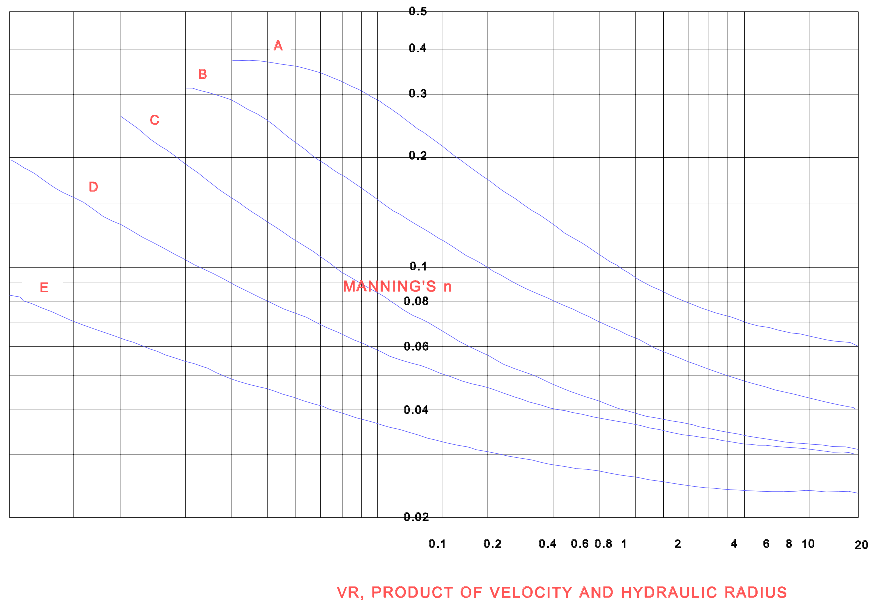


Figure 5. n-value relationships for grass cover.

Distribution of Hydraulic Roughness

Hydraulic roughness should be prescribed between each pair of coordinates, i.e., each panel, as shown in Figure 6. It is important to establish which portion of the channel cross section is bed and which is bank because the bed roughness predictors apply only to the channel bed. That is, typically the vegetation roughness and bank angle do not permit the bed load to move along the face of the banks. Therefore, the Limerinos and Brownlie n-value equations should not be used to forecast bank roughness, i.e., should not be assigned to the channel banks.

On the other hand, the point bar is a natural source-sink zone for sediment transport. Consequently, it is a location at which Limerinos and Brownlie equations would apply.

None of the n-value equations account for momentum or bend losses. Presently, the only technique to account for bend losses is to increase the n-values by some factor. Chow (1959) presents the Cowan method for including bend losses. This method requires the user to input the n-values directly, thus this method could be applied by the user to the SAM input.

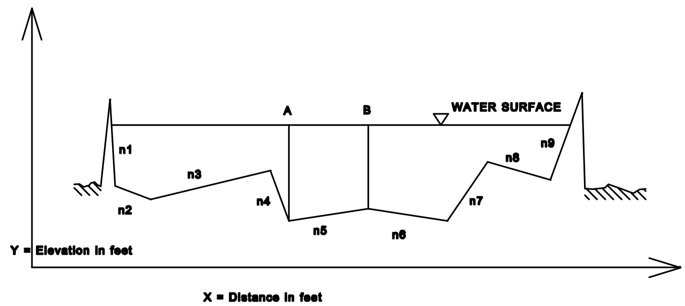


Figure 6. Prescribing hydraulic roughness.

Composite Hydraulic Properties

The compositing routines in SAM calculate representative hydraulic parameters for cross sections with complex geometry and variable roughness. Representative hydraulic parameters are used to calculate the same normal depth that would be calculated by a more rigorous analysis that analyzed each homogeneous cross-section subarea separately. Due to the obfuscated relationships between variables that can occur in compound and complex channels, parameter definition for a composite variable may be different than normally employed for that variable. For example, in the alpha compositing method, the composite hydraulic radius is not defined as the total area divided by the wetted perimeter; rather it includes, in addition to the usual geometric element property, the variation of both depth and n-values. There are several methods in the literature for compositing. The alpha method was selected as the default for SAM. Three other methods are provided as options: equal velocity, total force, and conveyance methods.

Alpha Method

The discussion of the derivation and use of hydraulic properties by the Alpha method appears in Appendix C, EM 1110-2-1601. A cross section is subdivided into panels between coordinate points. The divisions between panels are assumed to be vertical. The cross section is not subdivided between channel and overbanks for this calculation.

A water-surface elevation is either assigned or assumed, and calculations begin at the first wet panel in the cross section. The geometric properties of area and wetted perimeter are calculated, and the panel's hydraulic radius is calculated from the area and wetted perimeter. The assigned roughness for the panel is converted to a Chezy C, and a panel conveyance is calculated. Computations move panel by panel to the end of the cross section. The composite cross-sectional area is defined as the sum of all the panel subareas and is the true area. The composite velocity is defined as the total discharge divided by the area, conserving continuity. The composite hydraulic radius is conveyance weighted as given by the following equation:

$$\overline{R} = \frac{\sum_{i=1}^{k} R_i\, C_i\, A_i \sqrt{R_i}}{\sum_{i=1}^{k} C_i\, A_i \sqrt{R_i}}$$ **Equation 2-24**

where:

$\underline{R}$ = the composite hydraulic radius
k = the number of panels
i = the panel number
$R_i = A_i / P_i$
C_i = the panel Chezy roughness coefficient
A_i = the panel cross-sectional area
P_i = Wetted perimeter in wet panel i

The composite roughness coefficient, $\underline{n}$, is calculated using the composite $\underline{R}$ in the Manning equation.

$$\overline{n} = \frac{1.486\ \overline{R}^{\frac{2}{3}}\ S^{\frac{1}{2}} \sum_{i=1}^{k} A_i}{Q}$$ **Equation 2-25**

where:

S = energy slope
Q = total discharge

The alpha method ignores roughness on vertical walls, and renders roughness contributions negligible when there are steep side slopes. When significant roughness is contributed by vertical or steep side slopes, one of the other compositing methods should be employed.

Equal Velocity Method

A more rational compositing method for cross-sections with rough vertical walls or steep side slopes is the equal velocity method. It was proposed independently by both Horton and Einstein (Chow, 1959) and assumes that the velocity is equal in all panels. All hydraulic variables are calculated in the normal fashion except the Manning roughness coefficient, which is calculated using the following equation.

$$\bar{n} = \frac{\left(\sum_{i=1}^{k-1} P_i \, n_i^{1.5} \right)^{2/3}}{\left(\sum_{i=1}^{k-1} P_i \right)^{2/3}}$$

Equation 2-26

where

$\underline{n}$ = the composite n-value for the total section
n_i = n-value in wet panel i
k = number of panels

Since only wetted perimeter, and not hydraulic radius, appears in this equation, it is always well behaved. EM 1110-2-1601 discusses two other equal velocity methods. However, Horton's method was retained in SAM.hyd because of its simplicity. It is adequate for simple cross-section shapes, and it is programmable for complex cross-section shapes.

Total Force Method

This method was proposed by Pavlovskii, by Muhlhofer, and by Einstein and Banks (Chow, 1959). It is based on the hypothesis that the total force resisting the flow is equal to the sum of the forces resisting the flow in each panel. The resulting composite n-value is

$$\bar{n} = \frac{\left(\sum_{i=1}^{k} P_i \, n_i^{2} \right)^{1/2}}{\left(\sum_{i=1}^{k} P_i \right)^{1/2}}$$

Equation 2-27

where

P_i = Wetted perimeter in wet panel i
n_i = n-value in wet panel i
P = Total wetted perimeter in cross section

Conveyance Method

With the conveyance method, a composite roughness coefficient is calculated based on weighted conveyances in three subsections. The conveyance method separates the overbanks from the channel so the calculations can be confined to strips, or subsections within the cross section, having similar hydraulic properties. The conveyance for each subsection can be calculated and the values summed to provide the conveyance for the entire cross section. The Manning equation is used to calculate conveyance instead of the Chezy equation used in the alpha method. Composite hydraulic radius is calculated as total area divided by total wetted perimeter, rather than as a conveyance weighted parameter -- thus the compositing characteristics are concentrated in the Manning's roughness coefficient. Conveyance for the overbanks and channel are calculated in English units using the following equations:

$$K_{LOB} = \frac{1.486\ A_{LOB} \left(\frac{A_{LOB}}{P_{LOB}} \right)^{2/3}}{\frac{\sum_{i=1}^{i=LCB} P_i\ n_i}{P_{LOB}}}$$

$$K_{CH} = \frac{1.486\ A_{CH} \left(\frac{A_{CH}}{P_{CH}} \right)^{2/3}}{\frac{\sum_{i=LCB}^{i=RCB} P_i\ n_i}{P_{CH}}}$$

$$K_{ROB} = \frac{1.486\ A_{ROB} \left(\frac{A_{ROB}}{P_{ROB}} \right)^{2/3}}{\frac{\sum_{i=LCB}^{k} P_i\ n_i}{P_{ROB}}}$$

Equation 2-28

$$K_{TOTAL} = K_{LOB} + K_{CH} + K_{ROB}$$

where:

K = conveyance
LOB = Left overbank
CH = channel
ROB = right overbank
LCB = left channel bank
RCB = right channel bank

The composite hydraulic radius is calculated using the following equation:

$$\overline{R} = \frac{\sum_{i=1}^{k} A_i}{\sum_{i=1}^{k} P_i}$$

Equation 2-29

The composite Manning's roughness coefficient for the total channel is calculated using the following equation:

$$\overline{n} = \frac{1.486\ A_{TOTAL}\ \overline{R}^{2/3}}{K_{TOTAL}}$$

Equation 2-30

Compositing in a narrow-deep channel

The importance of accounting for the roughness of the side slopes in the calculation of normal depth is illustrated by the following example. A proposed trapezoidal channel has a 60 ft base width, 1V:2H side slopes, a slope of 0.001, and a design discharge of 5000 cfs. Assuming that the sides slopes have a roughness coefficient of 0.08 and the bed has a roughness coefficient of 0.030, normal depth and composited hydraulic parameters, calculated using the four compositing methods in SAM, are shown in the following tabulation:

The calculated depth is significantly lower with the alpha method because the side slope roughness is inadequately accounted for due to the assumption of vertical divisions between panels. If the side slopes had been vertical the alpha method would have completely neglected their contribution to roughness.

Comparison of Calculated Composite Hydraulic Parameters
Deep-Narrow Channel with High Side Slope Roughness

Method	Depth ft	Area sq ft	R ft	Velocity ft/sec	n
Alpha	10.4	839	10.0	6.0	0.037
Equal Velocity	14.7	1312	10.4	3.8	0.059
Total Force	15.0	1356	10.6	3.7	0.062
Conveyance	14.3	1262	10.2	4.0	0.056

Compositing compound channels and overbanks

Variable roughness and depth across the cross-section introduce turbulence into the flow which results in additional energy loss due to the momentum transfer. Existing theory is currently inadequate to properly quantify the magnitude of these losses. However, it is important that these losses be recognized and considered when choosing a compositing technique.

Compositing by the equal velocity and total force methods can overemphasize the contribution to roughness of channel subareas in cases where homogeneous flow conditions do not exist, such as on irregular overbanks. The same effect can occur with the conveyance method in SAM when only a single subsection is considered. James and Brown (1977) reported that without adjustments to either the resistance coefficient or the hydraulic radius, using composite hydraulic parameters in the Manning or Chezy equations did not accurately predict the stage-discharge relation in a channel-floodplain configuration when there were shallow depths on the floodplain--1.0 < Y/D < 1.4, where Y is water depth in the channel and D is the bank height. However, the effects of geometry seemed to disappear at the higher stages, i.e., for Y/D > 1.4, when it no longer became necessary to make any correction to the basic equations. Figure 7 summarizes that finding, stressing that the conveyance (separate-channels) method is the best choice if Y/D falls within the 1.0 to 1.4 range and that another method should be used if Y/D > 1.4.

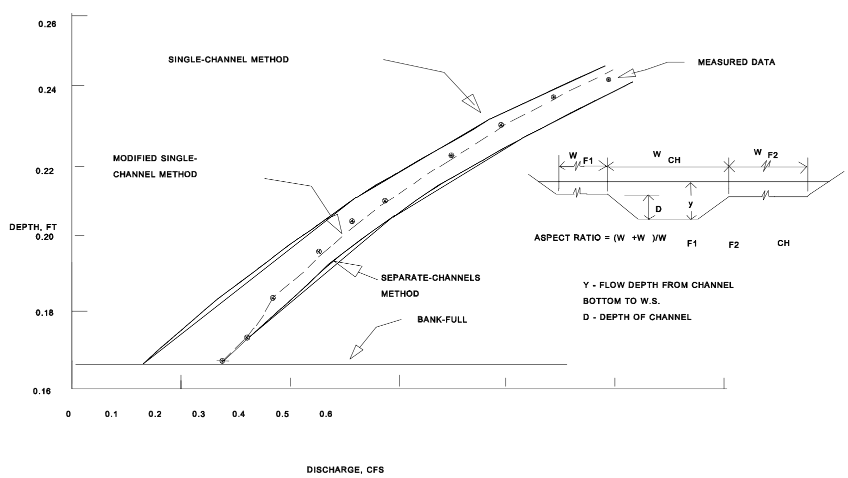


Figure 7. Comparison of measured data and theoretical methods.

Effective Hydraulic Parameters for Sediment Transport

The problem of obtaining representative hydraulic parameters is critical when making sediment transport calculations involving complex cross sections. The velocity, depth, width and slope are needed for subsections having similar hydraulic properties. This requirement leads to a compositing technique that produces an EFFECTIVE WIDTH and an EFFECTIVE DEPTH. Variables in the following equations are illustrated in Figure 8.

$$EFD = \frac{\Sigma D_i \ A_i \, D_i^{2/3}}{\Sigma A_i \ D_i^{2/3}}$$ **Equation 2-31**

$$EFW = \frac{\Sigma \ A_i \, D_i^{\ 2/3}}{EFD^{5/3}}$$ **Equation 2-32**

where

A_i = Area of panel i, $\Delta X \, D_i$
D_i = Average depth of panel i, 1/2 (A+B)
EFD = Effective Depth of the cross section
EFW = Effective Width of the cross section

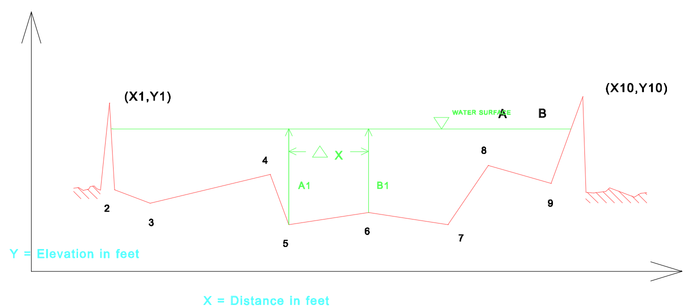


Figure 8. Calculating effective depths and widths.

In SAM the slope can be either the bed slope, water surface slope, or energy slope since all are parallel in normal depth computations. Effective values are calculated in SAM after the normal depth has been determined. Although effective values are calculated for both overbanks and the channel, only the channel values are written into the sediment transport input file for sediment transport calculations.

Flow Distribution Across the Cross Section

Flow distribution across the cross section will be calculated when all the variables in the uniform flow equation are prescribed. In addition, flow distribution will be calculated after the uniform flow equation is solved for an unknown variable. A conveyance weighted discharge, Q_i, is calculated for each panel using the following equation:

$$Q_i = Q\left(\frac{C_i\ A_i\ \sqrt{R_i}}{\Sigma\ C_i\ A_i\ \sqrt{R_i}}\right)$$ **Equation 2-33**

where

Q = total discharge
C = Chezy coefficient
A = area
R = hydraulic radius
i = panel number

Bottom Width Calculation--Fixed Bed

This option allows bottom width to become the dependent variable in the uniform flow equation:

$$W = f\ \ (Q, n, D, z, S)$$ **Equation 2-34**

where

Q = water discharge
D = water depth
z = side slopes of the channel
n = n-value
S = energy slope

This bottom width calculation is strictly a solution of the uniform flow equation for a fixed bed and does not consider sediment transport requirements. This option can only be used when a simple compound channel is specified on a CT-record. The compound channel may have up to 3 trapezoidal-shaped templates. The solution technique begins by sizing the low flow channel first, then solving for the main channel and then the overbank channel.

Energy Slope Calculation

This option allows the energy slope to become the dependent variable in the uniform flow equation:

$$S = f\ (Q, n, W, D, z) \qquad \textbf{Equation 2-35}$$

The calculations are trial and error and convergence may be difficult. The Brownlie n-value relationships may result in convergence failure due to the discontinuity at the transition between lower and upper regime flow. SAM.hyd prints a message when convergence fails.

Hydraulic Roughness Calculation

This option allows the n-value to become the dependent variable in the uniform flow equation.

$$n = f\ (Q, W, D, z, S) \qquad \textbf{Equation 2-36}$$

It is especially useful for calculating unknown roughness for one section of the cross-section when roughness in known for other sections of the cross-section.

This calculation, like the other solutions of the uniform flow equation which involve compositing, is trial and error. A simple solution of the Manning equation for the total cross section is used to calculate the first trial roughness. Roughness can be specified in some panels and calculated for others. If a roughness in a panel is input, the program will convert it to a k_s using the Keulegan equation for fully rough flow unless the equation specified is Strickler. However, only the Keulegan or Strickler equations can be used to calculate roughness. If the program is asked to calculate roughness using the Manning's equation option then it automatically uses the Strickler equation. If the program is asked to calculate roughness using the Brownlie or Limerinos equation then there is nothing to calculate because the roughness has been implicitly determined by the bed gradation used. If the program is asked to calculate roughness using the Grass equations then there is still nothing to calculate because the roughness is again implicitly determined by the Grass equation chosen.

NOTE: Limerinos, Brownlie and the Grass equations are not to be used with negative roughness values because there is nothing to calculate — the roughness is implicitly determined by the bed gradation or the grass.

Roughness in the remaining panels will be calculated as a specified proportion of k_s for the Manning, Strickler, and Keulegan equations. The iterative solution for k_s uses the secant method for convergence as follows.

$$X_3 = X_2 - \left(\frac{f(X_2)}{f(X_2) - f(X_1)} \right) (X_2 - X_1)$$ **Equation 2-37**

where

X_1 = is the first trial value of k_s

$f(X_1)$ = is the difference between Qtrue and the calculated Q for X_1

X_2 = is the second trial value of k_s

$f(X_2)$ = is the difference between Qtrue and calculated Q for X_2

X_3 = is the next trial value of k_s.

Of the several equations for hydraulic roughness, the Strickler equation is the most likely to converge. The Limerinos, Brownlie, and grass equations are independent of k_s and therefore cannot be used to solve for k_s. This algorithm is primarily intended for use with equations using k_s. Expect convergence problems with other equations.

Water Discharge Calculation

This option allows the water discharge to become the dependent variable in the uniform flow equation.

$$Q = f(Q, W, D, z, S)$$ **Equation 2-38**

This calculation, like the other solutions of the uniform flow equation which involve compositing, is trial and error. It also uses the secant method for convergence. When convergence fails, assume a range of discharges and use the normal depth calculations to arrive at the correct value.

Riprap Size for a Given Velocity and Depth

When flow velocity and depth are known, the riprap size is calculated using the following equation which taken from EM 1110-2-1601(1991,1994):

$$d_{30cr} = S_f\, C_S\, C_V\, C_T\, D \left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right)^{0.5} \frac{V_{AVE}\, C_B}{\sqrt{K_1\, g\, D}} \right]^{2.5} \qquad \textbf{Equation 2-39}$$

where

- d_{30CR} = Critical d_{30} (i.e. minimum d_{30}) size for stable riprap
- S_f = Safety factor
- C_S = Coefficient of incipient failure
- C_V = vertical velocity coefficient
- C_T = coefficient for riprap thickness
- D = local water depth
- γ_w = unit weight of water
- γ_s = unit weight of riprap
- V_{AVE} = average channel velocity
- C_B = bend correction for average velocity (V_{SS}/V_{AVE})
- K_1 = Correction for side slope steepness
- g = Acceleration of gravity

Required input data for simple riprap calculations are: average channel velocity, depth at the toe of the riprap, channel width, side slope, bend radius, bend angle, riprap specific weight, and whether the cross-section is trapezoidal or natural.

Riprap Gradation Tables

Size and specific gravity of available riprap are needed for this calculation, and the 13 standard riprap sizes shown in EM 1110-2-1601 are encoded into SAM.hyd as shown in the following table. Calculations begin with the smallest size stone and continue until a stable size is reached.

Quarry-Run Stone

A known riprap gradation table may be specified in SAM. This option is useful for quarry-run riprap where the gradation is known. Up to 5 sizes of quarry run riprap can be encoded. These gradations should be entered one size per record starting with the smallest and ending with the largest size. When this information is present, riprap size computations use the quarry run stone. When quarry run stone is prescribed, those stone sizes are used in lieu of the gradation tables encoded in SAM.

Table 6. Graded Riprap Sizes.

Low Turbulence zones, placed in the dry, γ_s = 165 lbs/cuft

Layer No.	DMAX[1] in	D30[1] ft	D50[1] ft	D90[1] ft	POROSITY[2] %
1	9	0.37	0.43	0.53	38
2	12	0.48	0.58	0.70	38
3	15	0.61	0.73	0.88	38
4	18	0.73	0.88	1.06	38
5	21	0.85	1.03	1.23	38
6	24	0.97	1.17	1.40	38
7	27	1.10	1.32	1.59	38
8	30	1.22	1.46	1.77	38
9	33	1.34	1.61	1.94	38
10	36	1.46	1.75	2.11	38
11	42	1.70	2.05	2.47	38
12	48	1.95	2.34	2.82	38
13	54	2.19	2.63	3.17	38

Notes:
[1] These values were taken from EM 1110-2-1601.
[2] These values are estimated from one set of field data.

Riprap Size for a Given Discharge and Cross Section Shape

Riprap size is a more complicated calculation when water discharge and cross section are given than it is when the flow velocity and depth are given because n-value becomes a function of riprap size. The computational procedure in SAM.hyd is as follows.

The computations begin with the unprotected channel. The bed sediment size is determined as the d_{50} calculated from the given bed gradation. Input must include hydraulic roughness equations and either n-values or k_s values, as required for normal depth computations. If the Strickler equation is selected for hydraulic roughness, the Strickler coefficient is 0.034, which is the value for natural sediment where $k_s = d_{50}$ (Chow, 1959). Normal depth is calculated using the alpha method, and flow is distributed across the section. Stability of the bed sediment is then calculated at each cross section coordinate using the distributed velocity and the depth at that point. Shield's Diagram is used to test for particle stability. The calculated shear stress is compared to the critical shear stress determined from Shield's diagram.

$$\tau_c = \Theta(\gamma_s - \gamma_w)\, d_{50}$$ **Equation 2-40**

where:

τ_c = critical shear stress
Θ = Shield's parameter

If this test shows that the actual bed shear stress exceeds the critical value then the bed sediment is diagnosed as unstable. If the panel is designated as a panel where riprap computations are desired, SAM will solve the riprap equation to find the smallest riprap size that will be stable. If the panel is not so designated, the program prints a message to the output file concerning the instability.

Considerable care must be employed when applying the cross-section shape option because flow conditions may be outside the range of conditions used to develop the riprap equations. When this option is used, it is important that the side slope protection be defined as a single panel in the geometry input. SAM will use 0.8 times the maximum depth in the panel for the local flow depth in the riprap equation. When a bend radius is specified, SAM uses the average velocity in the cross section for V_{AVE} in the riprap equation. Therefore, it is important that the input geometry include only channel geometry and discharge. When a bend radius is not specified, SAM uses the calculated panel velocity for V_{AVE} in the riprap equation. This calculation may provide useful information for compound channels in straight river reaches, but it is an extension of the procedure outlined in EM 1110-2-1601. Careful study of the recommendations and guidelines described in EM 1110-2-1601 should be considered essential.

In SAM, riprap computations begin with the smallest rock size. The hydraulic roughness equation, in each panel designated as having riprap, is automatically changed to the Strickler equation and the Strickler coefficient, is set equal to 0.034. Normal depth is calculated for the resulting n-values. The alpha method is used to calculate normal depth and flow is distributed across the section. The riprap size equation is solved for each panel. When the resulting size is stable in each panel, riprap computations are finished. Otherwise, computations move to the next larger riprap size and the procedure is repeated.

After the stable stone size is determined, a stage discharge curve is calculated for the riprapped channel. The Strickler coefficient, which was 0.034 when determining stone size, is increased to 0.038 in this calculation for flow capacity. This calculation determines the rating curve with the selected riprap in place.

Blench Regime Equations

Stable channel dimensions may be calculated using the Blench (1970) regime equations. These regime equations are also shown in ASCE Manual 54 (ASCE 1975). The equations were intended for design of canals with sand beds. The basic three channel dimensions, width, depth and slope, are calculated as a function of bed-material grain size, channel-forming discharge, bed-material

$$W = \left(\frac{F_B \; Q}{F_S} \right)^{0.5}$$ **Equation 2-41**

$$F_B = 1.9 \sqrt{d_{50}}$$ **Equation 2-42**

$$D = \left(\frac{F_S \; Q}{F_B^2} \right)^{\frac{1}{3}}$$ **Equation 2-43**

$$S = \frac{F_B^{0.875}}{\frac{3.63 \; g}{\nu^{0.25}} W^{0.25} D^{0.125} \left(1 + \frac{C}{2,330} \right)}$$ **Equation 2-44**

sediment concentration, and bank composition.
where

W = channel width - ft
F_B = bed factor
F_S = side factor
Q = water discharge -cfs
d_{50} = median grain size of bed material - mm
D = depth - ft
S = slope
C = bed-material sediment concentration - in parts per million
g = acceleration of gravity - ft/sec^2
υ = kinematic viscosity - ft^2/sec

The results are true regime values only if Q is the channel forming discharge. However, a width, depth and slope will be calculated for any discharge by these equations.

Blench suggests that the following values be used for the side factor:

F_S = 0.10 for friable banks
F_S = 0.20 for silty, clay, loam banks

F_S = 0.30 for tough clay banks

In order to calculate the Blench regime dimensions, the side factor, bed-material sediment concentration, and the bed-material gradation should be input. SAM.hyd sets default values for the Blench variables if none are specified: F_S = 0.20, C = 0.0, and d_{50} = 0.25 mm.

Stable Channel Dimensions

There is presently one analytical procedure in SAM for calculating stable channel dimensions. (Copeland, 1994) This analytical approach determines dependent design variables of width, slope, and depth from the independent variables of discharge, sediment inflow, and bed material composition. It involves the solution of flow resistance and sediment transport equations, leaving one dependent variable optional. Minimum stream power is used as a third equation for an optional unique solution. This method is based on a typical trapezoidal cross section and assumes steady uniform flow. The method is especially applicable to small streams because it accounts for transporting the bed material sediment discharge in the water above the bed, not the banks, and because it separates total hydraulic roughness into bed and bank components.

Basic Equations

The method uses the sediment transport and resistance equations developed by Brownlie (1981). There are separate resistance equations for upper and lower regime flow. The equations are dimensionless and can be used with any consistent set of units.

Upper regime:

$$R_b = 0.2836\, d_{50}\, q_*^{0.6248}\, S^{-0.2877}\, \sigma^{0.0813}$$

Equation 2-45

Lower regime:

$$R_b = 0.3742\, d_{50}\, q_*^{0.6539}\, S^{-0.2542}\, \sigma^{0.1050}$$

Equation 2-46

$$q_* = \frac{V\,D}{\sqrt{g\,d_{50}^3}}$$

where:

R_b = hydraulic radius associated with the bed
d_{50} = median grain size
S = slope
σ = geometric bed material gradation coefficient
V = average velocity
D = water depth
g = acceleration of gravity

To determine if upper or lower regime flow exists for a given set of hydraulic conditions, a grain Froude number F_g and a variable F_g' were defined by Brownlie. According to Brownlie, upper regime occurs if $S > 0.006$ or if $F_g > 1.25F_g'$, and lower regime occurs if $F_g < 0.8F_g'$. Between these limits is the transition zone. In the analytical method, $F_g = F_g'$ is used to distinguish between upper and lower regime flow. The program will inform the user which regime is being assumed in the calculations and if the bed forms are in the transition zone.

$$F_g = \frac{V}{\sqrt{g\, d_{50}\left(\frac{\gamma_s - \gamma}{\gamma}\right)}}$$

Equation 2-47

$$F_g' = \frac{1.74}{S^{0.3333}}$$

where: γ_s = specific weight of sediment
γ = specific weight of water

The hydraulic radius of the side slope is calculated using Manning's equation:

$$R_s = \left(\frac{V\, n_s}{1.486\, S^{0.5}}\right)^{1.5}$$

Equation 2-48

where: R_s = hydraulic radius associated with the side slopes, ft
V = average velocity, fps

n_s = Manning's roughness coefficient for the bank

If the roughness height k_s of the bank is known, then it can be used instead of Manning's roughness coefficient to define bank roughness. The model uses Strickler's equation to calculate the bank roughness coefficient:

$$n_s = 0.039\, k_s^{\frac{1}{6}} \qquad \textbf{Equation 2-49}$$

where k_s is the roughness height, in ft. For riprap, k_s should be set equal to the minimum design d_{90}.

Composite hydraulic parameters are partitioned in the manner proposed by Einstein (1950):

$$A = R_b P_b + R_s P_s \qquad \textbf{Equation 0-50}$$

where: A = total cross-sectional area
P_b = perimeter of the bed
P_s = perimeter of the side slopes

This method assumes that the average velocity for the total cross section is representative of the average velocity in each subsection.

Concentration, C, in ppm, is calculated using the Brownlie sediment transport equation, which is also a regression equation and is based on the same extensive set of flume and field data used to develop his resistance equations. This equation was chosen because of its compatibility with the resistance equations, which are coupled with the sediment transport equation in the numerical solution:

$$C = 9022\,(F_g - F_{go})^{1.978}\, S^{0.6601} \left(\frac{R_b}{d_{50}}\right)^{-0.3301} \qquad \textbf{Equation 2-51}$$

$$F_{go} = \frac{4.596\, \tau_{*o}^{0.5293}}{S^{0.1405}\, \sigma^{0.1606}}$$ **Equation 2-52**

$$\tau_{*o} = 0.22\, Y + 0.06\left(10\right)^{-7.7\, Y}$$ **Equation 2-53**

$$Y = \left(\sqrt{\frac{\gamma_s - \gamma}{\gamma}} \right)^{-0.6}$$

$$R_g = \frac{\sqrt{g\, d_{50}^{\,3}}}{\nu}$$

where
and

A typical cross section, with the critical hydraulic parameters labeled, is shown in Figure 9. The concentration calculated from the sediment transport equation applies only vertically above the bed. Total sediment transport in weight per unit time is calculated by the following equation:

$$Q_s = \gamma\, C\, B\, D\, V$$ **Equation 2-54**

where: Q_s = sediment transport in weight/time
B = base width

$$\overline{C} = \frac{Q_s}{0.0027\, Q}$$ **Equation 2-55**

An average concentration for the total discharge is then calculated:

where: _C = concentration using the total discharge in ppm
Q_s = sediment transport in tons/day

Q = discharge in cfs

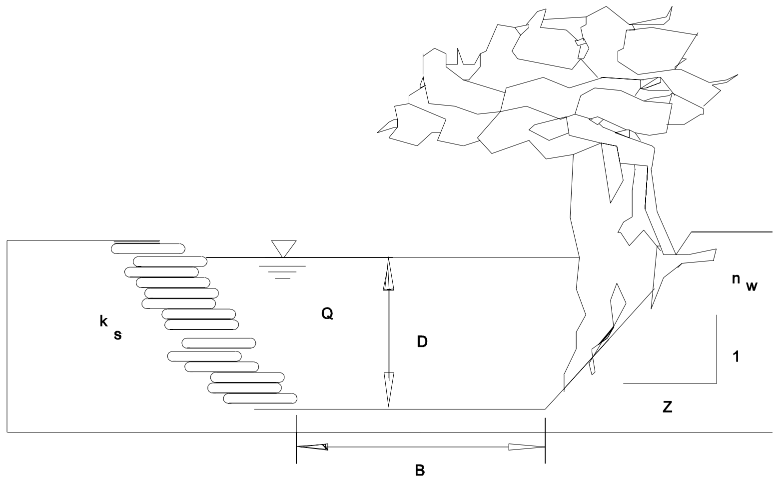


Figure 9. Typical cross section used in analytical method.

Input Requirements

Required input data are sediment inflow concentration, side slope, bank roughness coefficient, bed material d_{50}, bed material gradation coefficient, and water discharge. If sediment inflow is to be calculated, which is the recommended procedure, then additional data are required for the supply reach. These are: base width, side slope, bank roughness coefficient, bed-material median grain size, geometric gradation coefficient, average slope, and discharge. It is important that the base width be representative of the total movable-bed width of the channel. The bank roughness should serve as a composite of all additional roughness factors, i.e., channel irregularities, variations of channel cross-section shape, and the relative effect of obstructions, vegetation, and sinuosity. Only flow that is vertical above the bed is considered capable of transporting the bed material sediment load.

Water Discharge

The design discharge is critical in determining appropriate dimensions for the channel. Investigators have proposed different methods for estimating that design discharge. The 2-year frequency flood is sometimes used for perennial streams. For ephemeral streams the 10-year frequency is sometimes used. The “bankfull” discharge is sometimes suggested. Others prefer using the "effective" discharge, which is the discharge that transports the most bed material sediment. Currently, there is no generally accepted method for determining the channel-forming discharge. It is recommended that a range of discharges be used in the analysis to test sensitivity of the solution.

Inflowing Sediment Discharge

This is the concentration of the inflowing bed material load. It is best if SAM.hyd is allowed to calculate the sediment concentration based on hydraulic conditions in the sediment supply reach. The bed material composition is defined by the median grain size and the gradation coefficient.

Valley Slope

Valley slope is the maximum possible slope for the channel invert. This value is used in the test for sediment deposition. If the required slope exceeds the prescribed valley slope, the following message is printed:

```
>>>>MINIMUM SLOPE IS GREATER THAN VALLEY SLOPE -
THIS IS A SEDIMENT TRAP <<<<
```

Bank Slopes and Roughness

The analytical method assumes that all bed material transport occurs over the bed of the cross section and that none occurs above the side slopes. Therefore, the portion of water conveyed above the side slopes expends energy but does not transport sediment, making "Flow Distribution" an extremely important calculation. The input parameters for flow distribution are bank angle and bank roughness. The recommended procedure to use for this is discussed in Chapter 5 of EM 1110-2-1601. Any roughness input for the bed will be disregarded by the calculations as bed roughness is calculated using the Brownlie equations. For maximum transport of sediment, use the steepest bank angle allowed by bank stability requirements.

Range of Solutions

Stable channel dimensions are calculated for a range of widths. For each combination of slope and base width, a unique value of depth is calculated. This can be used to evaluate stability in an existing channel or in a proposed design channel. It is important to consider river morphology when interpreting these calculated values. It is also important to be consistent in the selection of channel dimensions. That is, once a width is selected, the depth and slope are fixed. This allows the designer to select specific project constraints, such as right-of-way or bank height or minimum bed slope, and then arrive at a consistent set of channel dimensions.

If the calculations indicate that the slope of the project channel needs to be less than the natural terrain, the slopes in the table can be used to aid in spacing drop structures or in introducing sinuosity into the project alignment.

An example of a family of slope-width solutions that satisfy the resistance and sediment transport equations for the design discharge is illustrated by Figure 11. Any combination of slope and base width from this curve will be stable for the prescribed channel design discharge. Combinations of width and slope that plot above the stability curve will result in degradation, and combinations below the curve will result in aggradation. The greater the distance from the curve, the more severe the instability.

Constraints on this wide range of solutions may result from a maximum possible slope, or a width constraint due to right-of-way. Maximum allowable depth could also be a constraint. Depth is not plotted in Figure 11, but it is calculated for each slope and width combination determined. With constraints, the range of solutions is reduced.

Different water and sediment discharges will produce different stability curves. First, the stable channel solution is obtained for the channel-forming discharge. Then, stability curves are calculated for a range of discharges to determine how sensitive the channel dimensions are to variations in water and sediment inflow events.

The stable channel dimensions are calculated for a range of widths on either side of a prescribed median value. If no median value is prescribed, the program assigns a value based on the hydraulic geometry equation proposed in EM 1110-2-1418.

$$B = 2.0\, Q^{0.5} \qquad \textbf{Equation 2-56}$$

The SAM program assigns 20 base widths for the calculation, each with an increment of 0.1B. Calculations for these conditions are displayed as output. Stability curves can then be plotted from these data.

A solution for minimum stream power is also calculated by the model. This solution represents the minimum slope that will transport the incoming sediment load. Solution for minimum slope is obtained by using a second-order Lagrangian interpolation scheme. Opinions are divided regarding the use of minimum stream power to uniquely define channel stability.

An optional use of the analytical method is to assign a value for slope, thereby obtaining unique solutions for width and depth. Typically there will be two solutions for each slope.

Meander Program

Langbein and Leopold (1966) proposed the sine-generated curve as an analytical descriptor for meander planform in sand-bed rivers. Their "theory of minimum variance" is based on the hypothesis that the river will seek the most probable path between two fixed points, which is described by the following equation:

$$\phi = \omega \cos \frac{2 s \pi}{M}$$ **Equation 2-57**

where:

- φ = angle of the meander path with the mean longitudinal axis
- ω = maximum angle a path makes with the mean longitudinal axis
- s = the curvilinear coordinate along the meander path
- M = the meander arc length

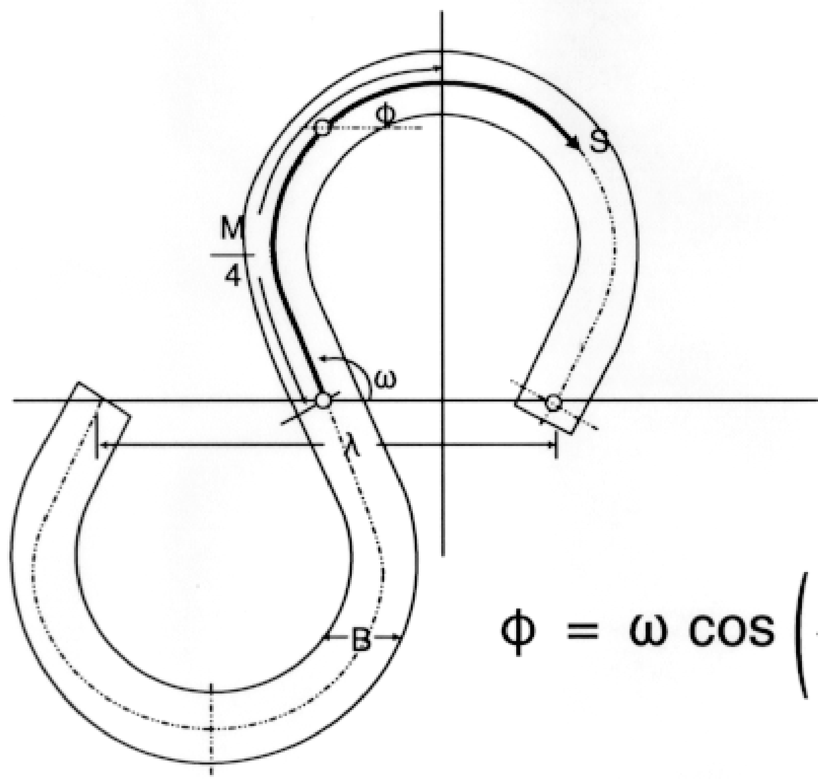
These variables are shown in Figure 10.

Meander planform may thus be described with a shape parameter, ω, and a scale parameter, M. The sine generated curve has been shown to effectively replicate meander patterns in a wide variety of natural rivers. (Langbein and Leopold, 1966)

The purpose of the meander algorithm in SAM is to provide both curvilinear and Cartesian coordinates for a meander planform based on the sine-generated curve. Required input are the meander arc length, M, and the meander wave length, λ. The meander arc length can be determined from the valley slope, the meander wave length, and the design stable channel slope.

$$M = \frac{\lambda \ * valley\ slope}{channel\ slope}$$ **Equation 0-58**

The stable channel dimensions of width, depth, and slope are determined using methods outlined in EM 1110-2-1418. The meander wavelength can be determined using hydraulic geometry relationships or analogy methodology.



$$\phi = \omega \cos\left(\frac{2\pi s}{M}\right)$$

Figure 10. Variable definitions for the Meander Program.

Figure 11. Graph of a family of slope-width solutions.